North Carolina Department of Transportation

Chapter 8 Bridges

January 2022

Revisions Sheet			
Page	**Old Section**	**New Section**	**Description**
-	-	-	• Entire Chapter revised to new format and minor grammatical changes made throughout • All references and links have been updated throughout Chapter
1	8.1	8.1	• Revised heading • 1st paragraph – revised wording
1	8.2	8.2	Entire section revised
1	8.7	8.3	Moved section; subsequent sections renumbered
2	8.3	8.4	Entire section revised
2	8.8	8.5	Moved section; subsequent sections renumbered; Entire section revised
5	8.5	8.7	4th bullet – revised to “Intermediate sections”
-	8.5.2.1	6.1	Moved to Chapter 6 - Resilience
6	8.5.2.2	8.7.2.1	Added last sentence
8	8.5.2.4	8.7.2.3	Last 2 sentences added
9	8.5.2.4	8.7.2.3	Figure 1 added (previously Appendix E – Item 6)
10	8.5.2.5	8.7.2.4	• Last sentence added • Added References to Figures 3 & 4 (previously Appendix E – Item 6)
10	8.5.2.6	8.7.2.5	Last bullet added
11	8.5.2.10	8.7.2.9	Entire section revised
14	8.5.2.12	8.7.2.11	Last sentence - removed "scanned and"
14	8.5.2.13	8.7.2.12	3rd sentence added - "Two-dimensional (2D) models are recommended for multiple opening analysis."
22	8.6.3	8.8.3	Added Figure 2 (previously Appendix R)
24	-	8.8.5	Added new section – Observed Scour Assessment Procedures
-	8.9	16.6	Moved to Chapter 16 – Coastal Hydraulic Design
-	8.6.5	16.6.1	Moved to Chapter 16 – Coastal Hydraulic Design

25	-	8.9	Added new section – References
28	-	8.10	Added new section – Additional Documentation
29	Appendix E – Item 5	8.10	Added Figure 3
30	Appendix E – Item 5	8.10	Added Figure 4

Table of Contents: Chapter 8, Bridges

8.1 Introduction

The primary goal of a bridge's hydraulic design is to set the bridge geometric constraints to convey floodwaters safely and efficiently without adversely affecting channel stability, the floodplain, the roadway facility, or adjoining properties. All Design Engineers should reference:

- Chapter 7 (Hydraulic Analysis for the Location and Design of Bridges) of the *AASHTO - Highway Drainage Guidelines* (AASHTO 2007)
- NCDOT *Bridge Policy* (NCDOT n.d.)
- FHWA *Hydraulic Design of Safe Bridges* (FHWA, L.W. Zevenbergen, L.A. Arneson, J.H. Hunt, A.C. Miller (authors) 2012)
- FHWA floodplain policy statement in *Federal Aid Policy Guide, 23 CFR 650A* (FHWA 1969)
- FHWA *Additional Guidance on 23 CFR 650A* (FHWA, J. Krolak 2011)

A bridge's design at a stream crossing requires a comprehensive engineering approach that involves data collection and documentation, hydrologic analysis, hydraulic analysis, scour evaluation, environmental impact evaluation, economic consideration, and documentation of the final design. The design procedures presented in this chapter will help ensure a systematic process that will adequately address most bridge crossing situations.

8.2 Priority for Consideration of Hydraulic Structure Type Recommendation

The recommended hydraulic structure type should be considered based on performance, cost, maintenance, and constructability.

8.3 Economic Consideration

When more than one alternative can satisfy all control factors for a site, the evaluation and selection of an optimal alternate should include a cost analysis to ensure that the selected alternate will be the most cost effective over the structure's life cycle.

8.4 Data Collection and Documentation

Information gathered during the pre-design study and field survey must be assembled for the study site. This process will include:

- Review of the Hydraulic Planning Report (Refer to Chapter 3), as well as available survey information
- Prepare preliminary bridge layouts sketches as appropriate and review with Division staff and the Structural Engineer.
- Prior to developing the final design, the following information should be gathered:
    - historical floods data, such as the high-water mark (elevation) and date of flood, etc.
    - existing and proposed features, such as utilities, road grades, drainage structures, bridge superstructure, bent locations, riprap armoring, etc.
    - adjacent structure elevation(s), such as the lowest adjacent grade and finished floor elevation of buildings, etc.
    - water surface elevation at date of survey and normal water surface (vegetation line, also known as ordinary high water) elevation
    - elevation of rock line from geotechnical subsurface investigation, if applicable
    - for coastal tidally influenced bridges, also show Mean High Water (MHW), Mean Tide Level (MTL), and Mean Low Water (MLW) elevations.
        - tidal datum information can be obtained from the National Oceanic and Atmospheric Administration (NOAA) website: www.noaa.gov
    - sufficient vertical clearance under bridge for maintenance and inspection activities
    - natural features, such as stream channel showing water's edge and top of bank, the existing land use and type(s) of vegetative cover in floodplain, jurisdictional streams, wetland limits, and riparian buffers
    - survey benchmark
    - manmade features in floodplain, such as buildings, houses, roads, utilities, levees, etc.

8.5 Documentation of Design

All information pertinent to the selection of the optimal final design alternate shall be documented in a manner suitable for review and retention, including:

- completion of the BSR (follow BSR key for additional guidance)

- show the proposed structure(s) and roadway grade in plan and profile, including crown grade elevation, superstructure, low chord, bent locations, limits of shoulder berm gutter (if applicable), riparian buffer zone (outer limit only, where applicable), specification of deck drainage accommodations, limits and elevations of rip rap and any channel modifications, typical bridge section, and any necessary details. In the bridge profile drawing, it is also important to show water surface elevations for Q_{design}, and Q_{100}. All water surface elevations should be expressed to nearest tenth of a foot (0.1 foot).
- in the BSR's interior gridded area, provide a performance table of the natural, existing (if applicable) and proposed conditions water surface elevations at the upstream toe section for the Q_{10}, Q_{design}, Q_{100}, and Q_{500} (or $Q_{overtopping}$, if less) discharges
    - specify the bridge face's distance upstream where the proposed conditions water surface elevations are referenced
- scour analysis computations on the BSR
- survey benchmark
- the following notation in the Additional Information and Computations section of the BSR, if applicable:
    - "No upstream or downstream structures that were in place at the time this project was designed will be adversely impacted by this bridge project."
- digital scan of sealed and approved BSR for digital archive record

copies of hydraulic computer model data files, with complete input and output, supporting (and consistent with) corresponding design documentation

- Information shown on the profile view includes, but is not limited to, the following:
    - centerline station, skew, structure (existing and proposed), span arrangement, lowest low chord, and natural ground (upstream and downstream) to accurately depict the floodplain and channel
    - existing bridge and piers should be shown with black dashed lines
    - In event of dual parallel bridges, separate profiles for each bridge may be needed. Inclusion of a typical section detail relating design grade point to centerline elevation is recommended.
- design and 100-year water surface elevations
- excavation in floodplain (note elevations), if applicable
- theoretical scour depths
- design scour depths (added later from geotechnical report when received)

  Plot estimated scour depths on the profile view for both the 100-year and 500-year return intervals (or for the overtopping discharge, if less, respectively)

Information shown on plan view includes, but is not limited to, the following:

- proposed structure

- existing structure in black dashed lines
- roadway superelevation
- limits of riprap for spill-through end bents
- for riverine flow, direction of flow and stream name
- in coastal tidal areas, direction of flood tide (landward/rising) and ebb tide (oceanward/falling)
- north arrow, stationing, and alignment
- floodway boundaries (for FEMA Detailed Study streams only)

  pertinent finished floor elevations on adjacent properties

8.6 Hydrologic Analysis

This phase involves the development of several discharges on which the performance of alternate designs will be evaluated. While the guidance in this chapter is intended to be specifically related to bridge design, much of the hydrologic analysis information presented here may be also applicable to culvert design.

The hydrologic analysis for bridge and culvert designs should include:

- determining a drainage area, land use, hydrologic region, etc. for the site
- developing flood discharges for a range of flood events (See Chapter 7 Hydrology)
- using Flood Insurance Study (FIS) discharges to evaluate compliance with FEMA regulations for a stream crossing that is in a FEMA FIS
- determining whether the FEMA discharges may be used for developing the hydraulic design
  - If there is considerable disparity between the FEMA study data and results from hydrological procedures presented in these *Guidelines*, the design engineer should determine the more appropriate method to use for developing the hydraulic design and document the justification for it on the BSR.

8.7 Hydraulic Analysis

This phase involves hydraulic analysis for review and selection of one or more alternatives. The bridge hydraulic design is typically based on a one-dimensional flow riverine step backwater analysis.

HEC-RAS is the most commonly used and widely accepted hydraulic modeling software (USACE 2021), (USACE 2021), (USACE 2021), (USACE 2021) to perform this type of analysis, and is the preferred software for most NCDOT bridge hydraulic design applications.

The Design Engineer should develop the HEC-RAS model with consideration of the following:

- utilize the cross-section configuration, as shown in Figure 5-1 of the HEC-RAS *Hydraulic Reference Manual* (USACE 2021).
- use a known starting water surface elevation the preferred downstream boundary condition for a subcritical run. Slope conveyance may be used if there is not a known starting water surface elevation.
- locate the beginning downstream section of the model at an adequate distance from the fully expanded flow section (Section 1, exit) to allow the step-backwater computations to converge to a correct water surface elevation before reaching Section 1.
- add intermediate sections to the model to ensure model stability.
- analyze all HEC-RAS hydraulic models as subcritical flow regardless of the channel gradient, unless use of alternate analysis method (e.g., supercritical flow or mixed flow) is approved by the State Hydraulics Engineer.
- use reliable historical flood data, if available, to calibrate the model.
    - publications FHWA TS-84-204 (USGS, G.J. Arcement, Jr., V.R. Schneider (authors) 1984) and USGS WSP 1849 (USGS, Harry M. Barnes Jr. (author) 1967) are good references for selecting Manning's roughness coefficient (n) values
    - use the HEC-RAS project file system to document all geometric, flow, and hydraulic design data configurations (plans) analyzed, including all water surface profiles, cross section plots, structures, and various output tables
    - HEC-RAS files submitted for approval should follow established naming and content conventions as specified on the Hydraulics Unit website
    - final design recommendations and supporting data from HEC-RAS should be appropriately documented on the BSR

Bridges in hydrodynamic, complex flow environments or tidally influenced areas may warrant utilization of unsteady or two-dimensional flow analyses methods, which are not discussed here. The design engineer should reference FHWA (FHWA, L.W. Zevenbergen, L.A. Arneson, J.H. Hunt, A.C. Miller (authors) 2012) for guidance and to obtain approval from the State Hydraulics Engineer before commencing design and analysis work using these methods.

8.7.1 General Design Criteria

Selecting an optimal final design alternative is accomplished by evaluating the study results with respect to acceptable design constraints, which are prescribed by both general and specific criteria.

- Avoid creating adverse impact of increased floodwater depth on properties upstream and downstream.

- Flow velocities through the hydraulic structure(s) should not result in channel instability or flood damage to the highway facility or adjacent property.
- Maintain existing channel and floodplain flow patterns to the extent practicable.
- Provide a level of traffic service that is consistent with the functional classification of the highway and projected traffic volumes to the extent practicable, unless a design variance is warranted.
- Project should result in minimal disruption of ecosystems and values unique to the floodplain and stream.
- Assess the floodplain impacts to properties during project construction, such as utilization of temporary causeway, temporary on-site detour and staging areas.
- Design pier and abutment location, spacing, and orientation to minimize flow disruption, debris collection and scour potential.
- Ensure compliance with National Flood Insurance Program regulations.

8.7.1.1 Sub Regional Tier Design

In 2008, NCDOT and FHWA approved guidelines establishing controlling design elements for new and reconstructed bridges on the state roads designated as minor collectors, local and secondary roads, which were published in the NCDOT document *Sub Regional Tier Design Guidelines for Bridge Projects* (NCDOT 2008). If a bridge project is designed to the standards set forth in that document, no formal design exception approval is required. The design engineer should read and become familiarized with these sub regional tier guidelines to ensure that an appropriate design process is followed.

8.7.2 General Design Criteria

8.7.2.1 Design Flood Frequency

This is the specific return period (frequency) flood that has been established as being an acceptable level for roadway overtopping. When roadway overtopping is not involved, it will be the level of flood used for establishment of freeboard and/or backwater limitations.

Overtopping is generally considered to be the point at which the computed water surface elevation overtops the minimum weir flow elevation. For bridge scour computations using HEC-RAS, begin computing weir flow when the energy grade line elevation exceeds the minimum weir flow elevation. Note when the energy grade line elevation is used as the basis for determination of when overtopping occurs in the BSR and in the modeling narrative, if applicable. See Chapter 7,Table 1 for desirable design discharge standards based on accepted inundation levels relative to roadway functional classification. Variation from these or other specific standard values must be justified by an assessment process which reflects consideration for risk of damage to the roadway facility and other properties, traffic interruption, cost, environmental impacts and hazard to the public. Generally, the design flood frequency should follow the Hydraulic Planning Report (HPR), unless otherwise approved by the Hydraulics Unit.

8.7.2.2 Backwater

Backwater is defined as the difference in the upstream water surface elevations between the non-encroached and encroached condition imposed upon the floodplain by the highway embankment and proposed structure. It is measured at the upstream toe of the roadway embankment. Backwater for the 100-year event should be limited to no more than one foot. If an existing structure already creates a 100-year backwater in excess of one foot, the design engineer may seek to replace it with a structure that reduces the backwater, provided it will not result in adverse impacts to the receiving channel and downstream properties. The backwater for the design year flood event for the proposed bridge should not exceed that of the existing bridge.

For National Flood Insurance Program (NFIP) regulated floodplains where no regulatory floodway has been established, the cumulative effect of the proposed highway encroachment combined with all other existing and anticipated development shall not result in backwater in excess of one foot above the established 100-year elevation shown on the effective FEMA Flood Insurance Rate Map (FIRM).

NCDOT’s policy is to compensate the adjoining property owners for the loss of their property value as the result of the proposed transportation facility. For example, an increase in floodway width would reduce a property owner’s developable land value.

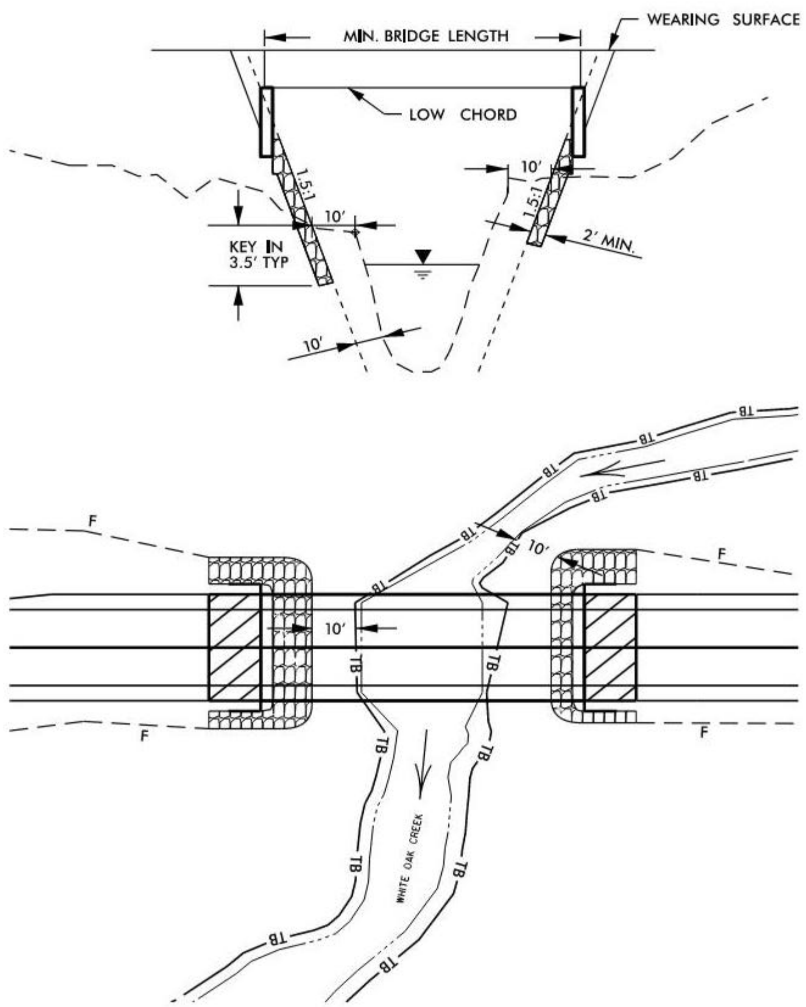
- Compensate, defined: to purchase or relocate the property, purchase floodplain (drainage) easement on the property, etc.

NCDOT follows the guidance provided in the 1982 Federal Highway Administration (FHWA) Memorandum of Understanding with the Federal Emergency Management Agency, entitled "Procedures for Coordinating Highway Encroachments on Floodplains within the Federal Emergency Management Agency (FEMA)", and the September 1992 FHWA NS 23 CFR Part 650A, Transmittal 5 (FHWA, J. Krolak 2011). When a detailed flood study area is involved and its regulatory floodway is established, typically no increase in backwater is allowed for the proposed conditions unless a Conditional Letter of Map Revision (CLOMR) proposal is developed and submitted to the community and FEMA for approval. A CLOMR proposal involves a revision in the floodway boundaries to accommodate the proposed transportation facilities without increasing the 100-year flood elevation above the established floodway elevation.

All potential CLOMR submittals for NCDOT projects must be reviewed by the State Hydraulics Engineer before submittal will be allowed to the respective regulatory agencies for approval. See Chapter 15 for guidance concerning FEMA NFIP compliance.

8.7.2.3 Minimum Length Bridge

For a bridge with spill-through abutments, the ends of the bridge should typically be located such that, anywhere along the abutment, a linear projection of the spill-through slope face normal to the direction of flow would provide a minimum of ten feet setback from any point on the channel bank or bed. The minimum length bridge is graphically depicted in Figure 1. Greater setback may be required due to the potential channel migration and scour prediction or other factors, such as greenway or animal passage accommodation. This does not necessarily preclude specification of a vertical abutment bridge, which could further reduce bridge length (which would eliminate the spill-through slope distance but would still require the ten-foot setback). Variances from the minimum setback are sometimes warranted, such instances should be discussed with the Hydraulics Unit. The final bridge length is determined by an appropriate hydraulic model during final design.



NCDOT MINIMUM LENGTH BRIDGE

Figure 1. NCDOT Minimum Length of Bridge

8.7.2.4 Bridge End Bent Cap

Generally, 4 feet end bent cap depths are used on new bridge designs. However, two feet, six-inch depth end bent caps may be a viable design option where warranted by site conditions, such as low roadway fill height. Figure 3 and Figure 4 shows two diagrams, which depict the dimensions for bridge waterway opening for both end bent cap depths. These diagrams should also be utilized to correctly specify the bridge waterway opening and minimum bridge rail (and guardrail) flow obstruction in a HEC-RAS hydraulic model and the associated bridge profile drawing in the BSR. The Bridge Engineer should be consulted to verify the final bridge end bent cap dimension.

8.7.2.5 Modeling Bridge Rail and Appurtenances

The Design Engineer should exercise judgment when coding in the bridge rail, guardrail, and any other appurtenances that may obstruct conveyance of flow (such as attached storm drain system or utilities). The following guidance is typically followed by convention for NCDOT projects but may not be applicable to every situation. The Design Engineer should document decisions to justify use of different methods or criteria than these in the modeling narrative. Model:

- Existing bridge rail based on height and length, and show as blocked
- Proposed bridge rail based on height and bridge length, and show as blocked
- At minimum, the first 12 feet of guardrail anchor unit at each end of the bridge and show as blocked (see Roadway Standard Drawings 862.03) (NCDOT 2012)
- Other appurtenances, such as an attached storm drain system or utility which may hang below the low chord of the bridge, thus reducing the waterway opening, using the bottom of the obstruction as the effective low chord
    - Note this clearly in the modeling narrative to specify the adjustments made to the low chord elevations to account for the obstruction
- Reference the MOA CCP document for further hydraulic modeling guidance

8.7.2.6 Substructure and Superstructure Determination

The bridge substructure components (drilled piers, piles, spread footings) are determined by the Geotechnical and Structures Management Units based on several factors such as subsurface soil data, loading requirements, navigational clearance, environmental constraints, etc. Early coordination with the Structures Management Unit is recommended at the beginning of the hydraulic design phase on decisions pertaining to the proposed bridge, such as superstructure type and depth, span arrangement, skew angle, longitudinal and cross slopes of deck, deck drainage, etc. Consideration should be given to the roadway overtopping flood level, freeboard, and potential impacts of raising the roadway grade. Piers should generally be aligned in the direction of flood flow. Span lengths and piers should be designed to minimize flow disturbance and drift traps as is consistent to good design and construction principles.

Prior to finalizing the design of a bridge, submit a draft copy of the BSR to the Structures Management Unit for comment.

8.7.2.7 Freeboard

Standard freeboard design for bridges shall be as follows:

- New location:
  - provide two feet minimum vertical clearance for bridge superstructure (low chord elevation) above the design flood elevation for primary route structures or secondary route crossings over major rivers
  - provide one-foot minimum vertical clearance for all other new location bridges (including temporary detour bridges).
- Existing location replacement:
  - If practicable, provide freeboard as stated above for new location crossings. Otherwise, as a minimum, maintain the freeboard provided by the existing bridge.

Greater freeboard may be needed for unique issues, such as heavy debris, climate change consideration, extreme weather (wind, storm surge), navigational clearance, etc. If the bridge deck is in superelevation, measure the freeboard at the low side of the low chord. It is also preferable, where practicable, that the low side of the superelevated bridge deck be set on the upstream side of the bridge. Variance from the freeboard requirement must be approved by the State Hydraulics Engineer prior to completion of the design.

8.7.2.8 Slope Protection

As a minimum, Class II rip rap should be placed on the spill-through abutment slopes through the waterway opening, extending beyond the bridge end bents along the roadway embankment 20 feet and 10 feet on the upstream and downstream sides, respectively. Along the roadway embankment, the top elevation of the rip rap should be placed either one foot above that of the design flood or up to the shoulder point elevation if the road is submerged during the design flood event, whichever is lower.

For a lake crossing, the elevation of the rip rap should be at least two feet above the normal pool elevation of the lake, or higher, if indicated by a wave run-up analysis. At the toe of fill, the rip rap protection should be keyed-in to a depth at least three and a half feet below the ground surface. Additionally, existing and potential stream bank erosion or instability should be considered, and riprap armoring should be provided as needed.

8.7.2.9 Bridge Deck Drainage

A minimum longitudinal gradient of 0.3% is recommended to facilitate adequate drainage of the bridge deck. For wide bridge decks and areas subject to debris buildup, a minimum 0.5% grade is recommended. When deck drains are needed, the typical design is:

- For girder-type bridges: specify six-inch (diameter) deck drains at twelve-foot centers on all girder-type bridges.
- For cored slab and box beam bridges: flow is discharged horizontally through the bridge rail via rectangular deck drains. The standard dimensions of these deck drain hydraulic openings are
    - eight inches wide by four inches high for cored slabs
    - five inches wide by four inches high for box beams
    - The actual drain opening is six inches high but will be obstructed by two inches of pavement overlay. These deck drains cannot be placed any closer than five feet (measured to center of the opening) from each end of the bridge or from either side of an interior bent and must have a minimum spacing of three feet (center to center).
- If the bridge is on a heavy skew, a minimum offset of six feet from the ends or interior bents of the bridge may be required. Deck drain capacity (and resulting spread calculations) should be evaluated assuming 30% blockage.

Consult the Structures Management Unit staff as early as possible in the design process regarding proposed deck drainage accommodations to verify constructability.

Examples of characteristics which may affect deck drainage could include, but are not limited to, the following:
- bridge type (girder, box beam, cored slab)
- deck drains are required for an entire span
- raised median on the bridge
- sidewalk
- barrier rail for protected bicycle/pedestrian lane included on bridge
- particular bridge rail type may affect deck drain locations

Collection of surface water at the end of the bridge could be needed regardless of usage of deck drains. When collection of surface runoff from the downgrade end of a bridge is needed, a grated drop inlet should be utilized. If there is inadequate depth for a grated drop inlet, a concrete flume may be used, extending to the toe of fill into a rip rap pad.

To the maximum extent practicable, bridge deck drains should not be placed directly over the stream. This is especially true for small streams and relatively short bridge lengths. For bridge spans requiring deck drains, the guidance in Chapter 13 should be followed regarding bridge crossings.

Further best practices regarding deck drains:

- avoid deck drains over spill-through rip rap abutments to reduce embankment erosion concerns
- provide rip rap pads beneath deck drains in highly erosive soil conditions. An armored shallow swale may be required in some instances to reduce erosion.
- enclose the drain system for a bridge deck:
  - If a closed drainage system is designed for a bridge deck, its outlet should be placed as far away as practicable from the protected surface water. A preformed scour hole is recommended at the outlet to help diffuse and infiltrate the stormwater unless other BMP devices are used.
  - Closed drainage systems are only specified for pre-stressed girder type bridges and will typically be comprised of 6-inch diameter PVC deck drains installed vertically through the deck connected to a longitudinal drainage system (typically an 8-inch diameter UVL-proof PVC pipe) beneath the deck.
  - To ensure positive drainage, a minimum 0.3% slope is desirable for the drainage system. Such closed systems are not desirable and should only be considered as a last resort if no other practicable alternatives are available.
- grade separation structures:
  - Bridges over roadways or railways shall not have deck drains which discharge directly over travel lanes, sidewalks, or railroad tracks. The gutter spread along the structure must be evaluated for issues affecting the safety of the traveling public, such as hydroplaning. This acceptable spread is dependent on shoulder or additional width provided on a structure, but generally should not extend into the through-travel lane (see Chapter 10, Section 10.3). Considering the potentially significant quantity of flow from the deck, it is very important to check the adequacy of the end drains and provide recommendations for additional measures when warranted.

The above guidelines must also be balanced with the safety need to limit the spread of storm runoff to minimize hazards such as hydroplaning and ice accumulation. (See guidance in Chapter 10, Section 10.3 and Table 10-1). Provision must be made at the down grade end of all bridges to adequately convey any storm runoff not intercepted by deck drains to a storm drain system or outlet. Further detailed guidance on bridge deck drainage design is provided in HEC-21 (FHWA, G.K. Young, S.E. Walker, F. Chang (authors) 1993).

8.7.2.10 Channel Relocation

The alignment of the proposed bridge and its piers should be designed to accommodate the existing channel. Prudent design consideration should be given to bridge crossings over unstable channels susceptible to high levels of bank erosion and channel migration. Repairing an unstable channel may be warranted to determine the proposed bridge length and location of end and interior bents. A major channel modification or relocation in and around a bridge crossing requires a thorough environmental assessment review, sound engineering design, cost analysis, and approval by the State Hydraulics Engineer.

8.7.2.11 Detour Structures

The design for a detour structure is site-specific. In general, the detour bridge and roadway grade should be designed to convey flood water during a ten-year flood event. These temporary structures may be lower and shorter than their permanent counterparts. They may result in potential risks, such as traffic interruption, flood damages to the roads and adjoining properties, etc. Generally, the detour bridges sit on two end bents that are supported by steel piles. The minimum length of a detour bridge is the width of the main channel plus a minimum of five-foot setback from each bank. On a site-by-site basis, the five-foot setback may be adjusted to ensure the integrity of channel banks and need of construction access. The bottom of the detour bridge deck (low chord) should allow at least one foot clearance over the flood elevation during the10-year flood event.

The theoretical scour analysis for the detour bridges may be limited only to the contraction scour during a ten-year flood event. For detour structures on FEMA-regulated streams, see additional guidance in Chapter 15, Section 15.6. When developing the detour bridge design, the Division Bridge Construction Engineer should be consulted regarding the potential type of temporary detour bridge structure anticipated to be utilized for the project.

Detour Survey and Hydraulic Design Report should be used to document design criteria used for detour bridges. Sketch proposed structure(s) and roadway grade in plan and profile showing roadway grade elevation, minimum low chord elevation, structure location and size, limits and elevations of any required scour protection (if applicable), and any channel modifications. These should be compiled into a single document to be distributed and filed appropriately.

8.7.2.12 Multiple Bridge Openings

Roadways over streams or rivers with wide floodplains may warrant multiple openings in the floodplain to provide better conveyance through the embankment. Whenever multiple openings are required, the design engineer should develop hydraulic models to assess the location and performance of each hydraulic opening structure. Two-dimensional (2D) models are recommended for multiple opening analysis. The results of the modeling and performance of these structures should be documented in the BSR. The guidance outlined in the *HEC-RAS Hydraulic Reference Manual and the HEC-RAS Two-Dimensional Modeling User's Manual* (USACE 2021) (USACE 2021) should be utilized.

8.8 Bridge Scour Evaluation

An estimate of potential scour depth is required for all new bridge designs. FHWA has issued a set of three Hydraulic Engineering Circulars (HECs) to provide guidance for bridge scour and stream stability analyses:

- HEC-18 *Evaluating Scour at Bridges* (FHWA, L.A. Arneson, L.W. Zevenbergen, P.F. Lagasse, P.E. Clopper (authors) 2012)
- HEC-20 *Stream Stability at Highway Structures* (FHWA, P.F. Lagasse, L.W. Zevenbergen, W.J. Spitz, L.A. Arneson (authors) 2012)
- HEC-23 *Bridge Scour and Stream Instability Countermeasures* (FHWA, P.F. Lagasse, P.E. Clopper, J.E. Pagán-Ortiz, L.W. Zevenbergen, L.A. Arneson, J.D. Schall, L.G. Girard (authors) 2009)

Bridge scour evaluation requires a multidisciplinary analysis that involves input from the design engineer, the Geotechnical Engineering Unit and the Structures Management Unit.

The design engineer's role in evaluating scour involves the following three steps:
1. Stream stability and geomorphic assessment
2. Scour analysis
3. Bridge scour and stream instability countermeasures

8.8.1 Stream Stability and Geomorphic Assessment

The Design Engineer should evaluate the stream stability and make a geomorphic assessment of the stream crossing. This part of the process includes office data collection, a site visit evaluation and an overall assessment of the stream stability. This information must be documented and will be used in the overall scour evaluation.

Office data collection specific to the scour evaluation includes but is not limited to:
- bridge routine inspection reports
- historical bridge survey reports
- FHWA Scour Program reports
- aerial photography
- old structure plans
- available geotechnical information

Information collected specific to the scour evaluation during the site visit includes but is not limited to:
- stream characteristics
  - straight, meandering, braided, anastomosed, engineered
- floodplain characteristics
  - natural, agricultural, urban, suburban, rural, industrial etc. and susceptibility to change

- overall stream stability:
  - lateral stream stability (plan form)
    - bank material, bank slope, bank vegetation, bank erosion, leaning trees, debris potential, floodplain material. Any past or possible channel migration should be noted.
  - vertical stream stability (profile)
    - bed material, degrading, aggrading, stable, scour holes, pools, riffles, etc.
  - stream response
    - stable or subject to change
- debris potential
  - leaning or undercut trees along banks, size and quantity
- scour at existing bridge to be replaced, if applicable

  observed conditions around existing piers and spill-through slopes, indication of previous scour, depth etc., foundation type – is footing visible?

Based on the above evaluations, the Design Engineer should make an overall assessment of the stream stability. In particular, the design engineer should note the potential for lateral shifting (migration) of the channel when evaluating scour of piers and or abutments close to the channel banks. Potential for lateral shifting (migration) of the channel should be considered in the layout of the bridge (location of piers and/or ends of bridge). See following guidance for calculating pier scour and abutment scour. A statement addressing the overall assessment of the stream stability and its determination in the scour evaluation should be noted on the BSR with the scour computations.

8.8.2 Scour Analysis

Evaluate scour design flood frequency as follows:

- Regional Tier and Statewide Tier Projects
  1. If the overtopping flood is less than the 100-year flood, analyze scour for the overtopping flood only. Show and plot overtopping scour calculations on the Bridge Survey Report.
  2. If the overtopping flood is greater than the 100-year flood but less than the 500-year flood, analyze scour for the 100 year and overtopping floods. Show and plot both scour calculations on the Bridge Survey Report.
  3. If the roadway is not overtopped by the 500-year flood, analyze scour for both the 100-year and 500-year floods. Show and plot both scour calculations on the Bridge Survey Report.

- Sub Regional Tier Projects
    1. If the overtopping flood is less than the 100-year flood, analyze scour for the overtopping flood only. Show and plot overtopping scour calculations on the Bridge Survey Report.
    2. If the overtopping flood is greater than the 100-year flood, analyze scour for the 100- year flood only. Show and plot 100-year scour calculations on the Bridge Survey Report.

8.8.2.1 Contraction Scour

The Design Engineer should evaluate contraction scour for all bridges. Normally, NCDOT bridge length provides a minimum ten foot setback from any point on the channel bank or bed, as described in Chapter 3, Section 3.5. Standard practice is to use spill-through sloped abutments lined with Class II rip rap keyed into the overbank area under the bridge a minimum depth of 3.5 feet. This is described as contraction scour Case 1c in HEC-18. Contraction scour typically should only be computed for the main channel and not the overbank areas between the main channel and the abutments. However, computing overbank contraction scour may be appropriate for a bridge spanning a very wide floodplain.

Live-bed contraction scour occurs at a stream when there is transport of bed material from the upstream reach into the bridge cross section. With live bed contraction scour, the area of the contracted section increases until a state of equilibrium occurs, at which the transport of sediment out of the contracted section equals the sediment transported in (FHWA, L.A. Arneson, L.W. Zevenbergen, P.F. Lagasse, P.E. Clopper (authors) 2012).

Clear-water contraction scour occurs when:

- there is no bed material transport from the upstream reach into the downstream reach, or
- the material transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of flow.

With clear-water contraction scour, the area of the contracted section increases until, in the limit, the velocity of flow or the shear stress on the bed is equal to the critical velocity or the critical shear stress of a certain particle size in the bed material (FHWA, L.A. Arneson, L.W. Zevenbergen, P.F. Lagasse, P.E. Clopper (authors) 2012).

Design guidance for calculating contraction scour is as follows:

- Determine if the scour design flood frequency water surface elevation results in non-pressure flow scour conditions (water surface elevation is below the low chord elevation of the bridge) or pressure flow scour conditions (water surface elevation is above the low chord elevation of the bridge).
- For non-pressure flow scour conditions, calculate contraction scour using the live bed contraction scour equation 6.2 in Chapter 6 of HEC-18 with a k_1 exponent of 0.69. The equation is:

$y_2 / y_1 = (Q_2 / Q_1)^{6/7} (W_1 / W_2)^{k_1}$

$y_s = y_2 – y_0$ = (average contraction scour depth)

Where:

y_1 = Hydraulic depth in the upstream main channel, ft.

y_2 = Hydraulic depth in the contracted section channel, ft. (this is computed by the equation)

y_0 = Hydraulic depth in the contracted section channel before scour (Use the upstream internal bridge section in HEC-RAS), ft.

Q_1 = Flow in upstream main channel, ft^3/s

Q_2 = Flow in contracted channel (use the upstream internal bridge section in HEC-RAS), ft^3/s

W_1 = Top width of upstream main channel, ft. (See note 4 in HEC-18 Section 6.3)

W_2 = Top width of main channel in contracted section (use the upstream internal bridge section in

HEC-RAS), ft.

k_1 = 0.69 (for worst case scenario)

To ensure accuracy of bridge contraction scour computations, the values of y_1, Q_1 and W_1 of the upstream main channel to be used in the contraction scour equations should be taken at the upstream approach section (fully effective unconstricted section). The approach section must be properly located and the channel geometry correctly verified by field surveys. The approach section should be located at a point upstream of the bridge just before the flow begins to contract due to the bridge opening.   This may require adding another upstream section in developing the Corrected Effective HEC-RAS model, especially in the case of Limited Detailed Study models, which may have been created with an upstream approach section that is not within a reasonable distance upstream to correctly represent the location at which flow contraction begins. It also may have only an approximated channel configuration not based on field surveys. In some instances, the channel width and floodplain geometry at the approach section may be considerably different than the channel nearer the bridge, in which case it would not be appropriate to use the approach section geometry for the contraction scour calculation. If this is the case, then the values of y_1, Q_1 and W_1 may be taken from the upstream toe section of the natural conditions model at the bridge location.

The Design Engineer should also carefully identify the channel section through the internal bridge opening. The top of bank stations should accurately define the channel through the bridge opening in the HEC-RAS model.

Non-pressure flow contraction scour conditions for overflow bridges should be calculated using clear water contraction scour equation 6.4 in chapter 6 of HEC-18. The equation is:

$y_2 = [(K_u Q^2) / (D_m^{2/3} W^2)]^{3/7}$

$y_s = y_2 - y_0$ = (average contraction scour depth)

Where:

y_2 = Average equilibrium depth in the contracted section after scour, ft.

Q = Discharge through the bridge associated with the width W, ft^3/s

D_m = Diameter of the smallest non-transportable particle in the bed material ($1.25D_{50}$) in the contracted section, ft.

D_{50} = Median diameter of bed material, ft.

W = Top width of the contracted section less pier widths, ft.

y_0 = Hydraulic depth in the contracted section channel before scour (use the upstream internal bridge section in HEC-RAS), ft.

K_u = 0.0077 for English Units

If the D_{50} bed material for the overflow bridge is not known, use D_{50} for very coarse sand (.007ft.). If the overflow bridge is part of a braided river system, the design engineer should use the live bed contraction scour equation.

Pressure flow scour conditions should be calculated as outlined in section 6.10 of chapter 6 of HEC 18. NCDOT practice is to only compute pressure flow scour conditions up to the point of roadway overtopping. Therefore Q_{ue} (effective channel discharge for live bed conditions and overtopping flow) is not required to be computed. The pressure flow scour equations should be used with the live bed contraction scour equation and/or the clear water contraction scour (for overflow bridges) as noted above.

The pressure flow scour equations are as follows:

$y_s = y_2 - h_b$

Where:

y_s = pressure flow scour depth, ft

y_2 = average depth in the contracted section as determined from the live bed contraction scour equation or contraction scour equation noted above, ft.

h_b = vertical height of bridge opening (bed to low chord) prior to scour, ft.

Contraction scour at bottomless culverts ("three-sided") is not required since NCDOT requires that these be founded on scour resistant rock.

8.8.2.2 Pier Scour

Evaluate pier scour for all internal piers. The design engineer should reference Equation 7.3 of HEC-18 to compute the pier scour as shown below:

$$y_s / a = 2 K_1 K_2 K_3 (y_1/a)^{0.35} Fr_1^{0.43}$$

Where:

y_s = Scour depth, ft.

y_1 = Flow depth directly upstream of the pier, ft (use the upstream toe section in HEC-RAS).

K_1 = Correction factor for pier nose shape from figure 7.3 and table 7.1 in HEC-18

K_2 = Correction factor for angle of attack of flow from table 7.2 or equation 7.4 in HEC-18

K_3 = Correction factor for bed condition from table 7.3 in HEC-18

a = Pier width, ft

L = Length of pier, ft

Fr_1 = Froude Number directly upstream of the pier=$V_1/(gy_1)^{1/2}$

V_1 =Mean velocity of flow directly upstream of pier, ft/s.

g = Acceleration of gravity (32.2 ft/s^2)

For complex pier foundations, the Design Engineer should use the procedures outlined in HEC-18. An Excel spread sheet developed for Florida DOT (FDOT) is also available for use in calculating complex pier foundations. It can be downloaded from FDOT's website.

Based on the stream stability and geomorphic assessment of the bridge site, a note should be added on the BSR with the pier scour calculations stating whether or not the local pier scour was calculated based on potential channel migration or no channel migration. If there is potential for channel migration such that the channel could migrate to the pier location, then the pier scour should be calculated based on the depth of flow from the channel bottom prior to scour. If there is no potential for channel migration, then the pier scour should be calculated based on the depth of flow at the pier location prior to scour.

8.8.2.3 Abutment Scour

Evaluate abutment scour for all vertical abutment bridges or spill-through abutment bridges that have less than the minimum ten-foot setback from any point on the channel bank or bed as noted above in 0. Abutment scour evaluation is not required for spill through bridges that are designed based on the minimum bridge length or greater unless it is determined through the overall assessment of the stream stability that abutment scour may be a concern.

The NCHRP 24-20 *Estimation of Scour Depth at Bridge Abutments* (NCHRP, R. Ettema, T. Nakato, M. Muste (authors) 2010) method outlined in Chapter 8 of HEC-18 should be used. It should be noted that the NCHRP 24-20 method calculates both abutment and contraction scour. The equations and procedure are as follows:

$$y_{max} = \propto_A y_c$$

$$y_c = y_1(q_2/q_1)^{6/7}$$

$$y_s = y_{max} - y_0$$

Where:

y_{max} = Maximum flow depth resulting from abutment scour, ft.

y_c = Flow depth including live bed contraction scour, ft.

$\propto_A$ = Amplification factor for live bed conditions.

y_1 = Hydraulic depth in the upstream (approach) main channel, ft.

q_1 = Upstream unit discharge, ft^2/s. Estimate by dividing the upstream channel discharge by the upstream channel top width.

q_2 = Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s. Estimate by dividing the total bridge opening discharge by the total bridge opening width.

y_s = Abutment scour depth, ft.

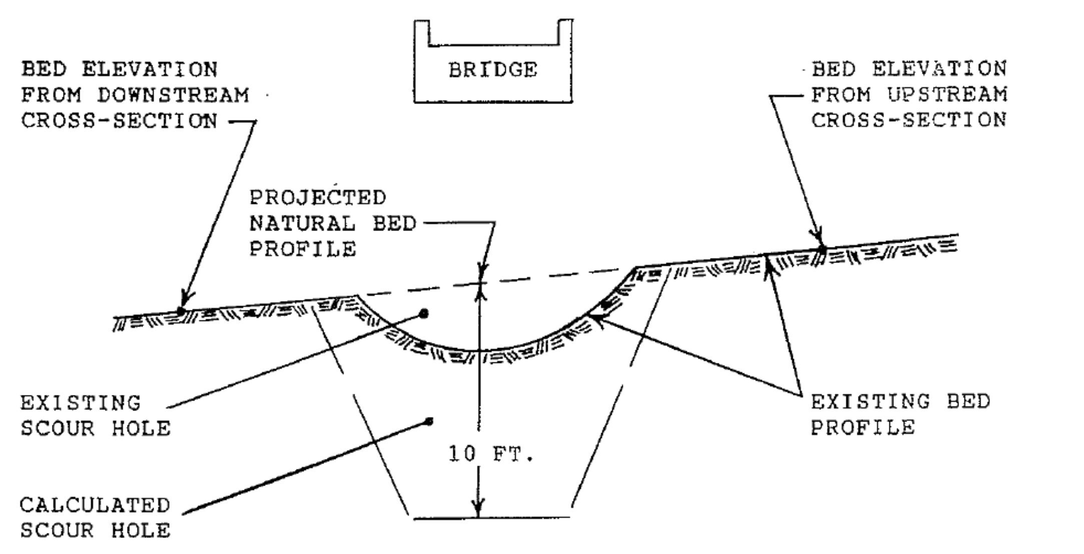
y_0 = Flow depth prior to scour, ft.

After calculating q_2 / q_1, the design engineer should use Figures 8.9 and 8.10 of HEC-18 to compute $\propto_A$. The values of y_c, y_{max} and y_s may then be calculated based on the equations above.

Froehlich's Abutment Scour Equation or the HIRE Abutment Scour Equation as outlined in HEC-18 may be used if determined to be more applicable and approved by the reviewing engineer.

8.8.3 Plotting Scour

The cone of influence (scour hole side slopes) for total scour to be shown on the bridge profile view of the BSR should be at least 1.4 H: 1 V. However, Section 7.8 of HEC-18 suggests using 2 H: 1 V (FHWA, L.A. Arneson, L.W. Zevenbergen, P.F. Lagasse, P.E. Clopper (authors) 2012). If only contraction scour is calculated, the design engineer may plot scour depth from channel bottom prior to scour. The width of the bottom of the contraction scour should match the channel bottom width. If there is an existing scour hole under the existing bridge, do not add the calculated scour depth to the existing scour depth, unless the existing scour depth was used in the y_2 calculation of scour and in the bridge hydraulic analysis. Show the depth of calculated scour relative to the projected natural stream bed; an example is illustrated in Figure 2.



EXAMPLE SHOWN IS FOR A TOTAL CALCULATED SCOUR DEPTH (CONTRACTION + PIER) EQUAL TO 10 FEET. THE SCOUR DEPTH IS MEASURED FROM THE PROJECTED NATURAL BED PROFILE, NOT FROM THE BOTTOM OF THE EXISTING SCOUR HOLE.

Figure 2. Diagram of Bridge Scour Depth Relative to Projected Natural Stream Bed

Based on the location of piers, the theoretical scour may be plotted as follows:

- if pier is in main channel:
  - add contraction and pier scours as the maximum scour and plot it below the thalweg elevation at the pier location
  - depth of flow and velocity for pier scour should be based on channel bottom elevation prior to scour
  - width of the bottom of the pier scour should be the width of the pier
  - plot both the side slopes of the pier and the contraction scours at 1.4:1
- if pier is not in main channel, but may be later due to channel migration:
  - add contraction and pier scours as the maximum scour at the pier location and plot it from the thalweg elevation
  - depth of flow and velocity for pier scour should be based on channel bottom prior to scour
  - width of the bottom of the pier scour should be the width of the pier
  - plot both the side slopes of the pier and the contraction scour at 1.4:1
- if pier is not in main channel with little potential for migration:
  - plot contraction scour as noted above
  - plot pier scour based on depth of flow at pier location prior to scour
  - if cone of influence of contraction scour intersects pier location below natural ground at pier, plot pier scour from this point
  - width of the bottom of the pier scour should be the width of the pier
  - plot both the side slopes of the pier and the contraction scours at 1.4:1
- abutment scour
  - use NCHRP 24-20 Method to plot the abutment scour
  - begin the plot of the scour at the lowest point in the stream bed out to the ends of the bridge end bents
  - note that the NCHRP 24-20 Method computes both contraction and abutment scour
  - if Froehlich's or Hire Abutment scour equations are used, plot abutment scour from ground elevation at abutment

8.8.4 Documentation of Scour on the BSR

The Design Engineer should include the following information in the "Additional Information and Computations" section of the BSR:

- overall assessment of the stream stability and its determination in the scour evaluation
- if pier is subjected to potential channel migration
- appropriate scour computations during each flood event
- evidence of existing scours in and around the main channel, interior and end bents

The Design Engineer calculates the theoretical scour based on the guidelines outlined in this section. This information must be documented on the BSR, which is provided to the Geotechnical Engineering Unit for their use in developing the Design Scour Elevations. Based on the Geotechnical Engineering Unit's Subsurface Investigation Report, the Design Scour Elevation may be adjusted from the Theoretical Scour

Elevation on the BSR. The Geotechnical Engineering Unit and/or the Structures Management Unit may consult with the Hydraulics Engineer throughout the scour evaluation process as necessary (NCDOT 2021).

8.8.5 Observed Scour Assessment Procedures

Follow the following procedures:

1. Observed scour issues will be reported in an email stating the structure number and a description of field observations (include pictures or a copy of the inspection report) to the Scour Team at ScourNotify@ncdot.gov.
2. The Hydraulics Unit develops a response to the reported scour issue through a review of all available documents and data related to the structure. The Hydraulics Unit will also suggest if the NBIS Item 113 Scour Code needs to be updated to reflect the current scour conditions. The response will be provided on a Scour Evaluation Form which details the scour issue, corresponding POA, and proposed item 113 code.
3. The Hydraulics Unit emails the Scour Evaluation Form to the representatives on the Scour Committee (includes members from Geotechnical, Hydraulics and Structure Management Units).
    - If there is a suggested change to NBIS item 113 or additional input is needed from Geotechnical and or Structures Management, the representatives on the Scour Committee provide feedback and concurrence.
    - If there is no suggested change to NBIS item 113 and additional input is not required, the representatives on the Scour Committee can provide comments if needed.
4. Once the Scour Evaluation Form has been finalized, the Hydraulics Unit will email the final report to the representatives on the Scour Committee, Inspectors, SIA, and other interested parties.

The completed version of this document will be stored in the structure file via WIGINS for record keeping.

8.9 References

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8.10 Additional Documentation

Bridge Survey & Hydraulic Design Report (BSR) Key

Detour Structure Survey & Hydraulic Design Report

MOA CCP document

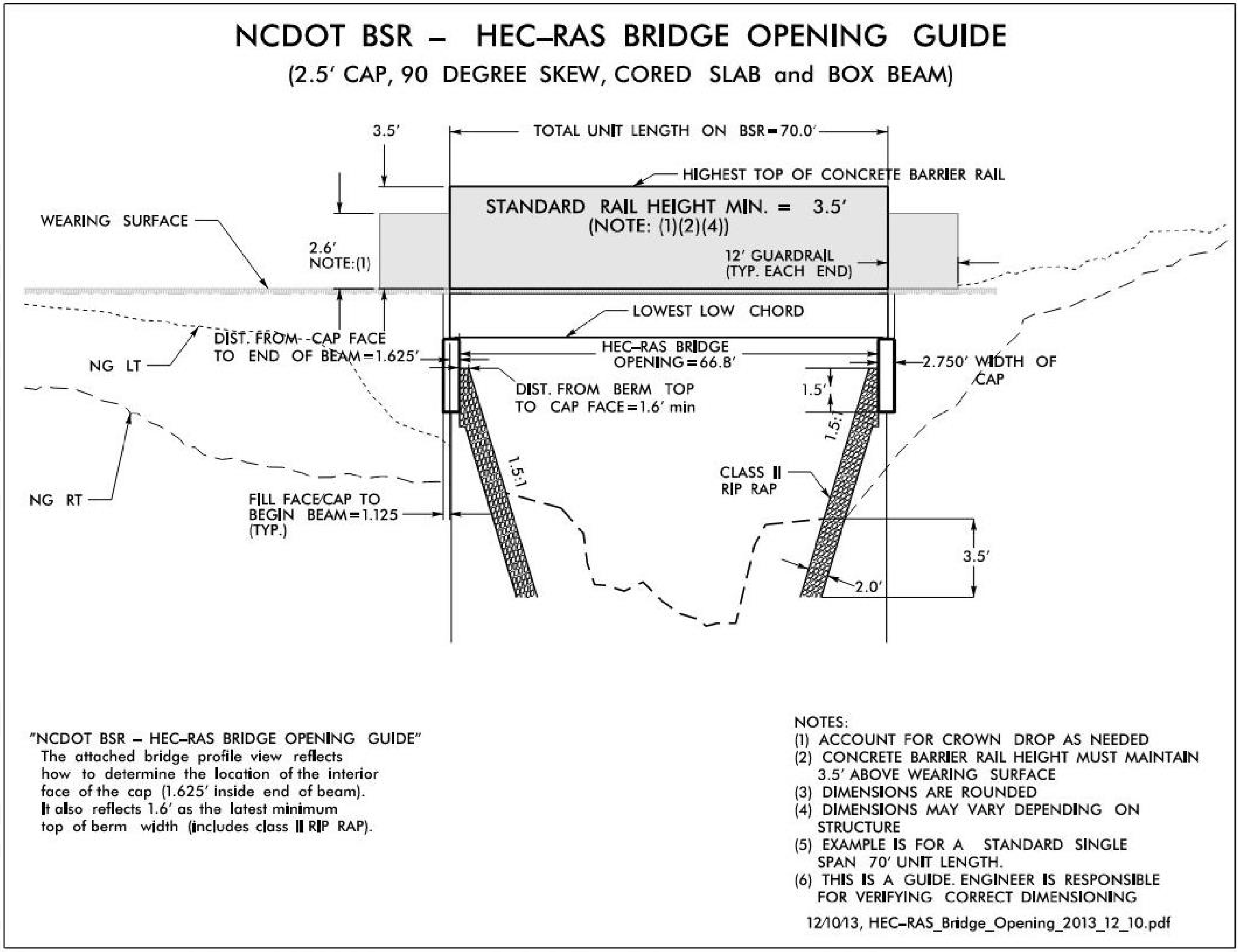

Figure 3. HEC-RAS Bridge Opening Guide (2.5' Cap)

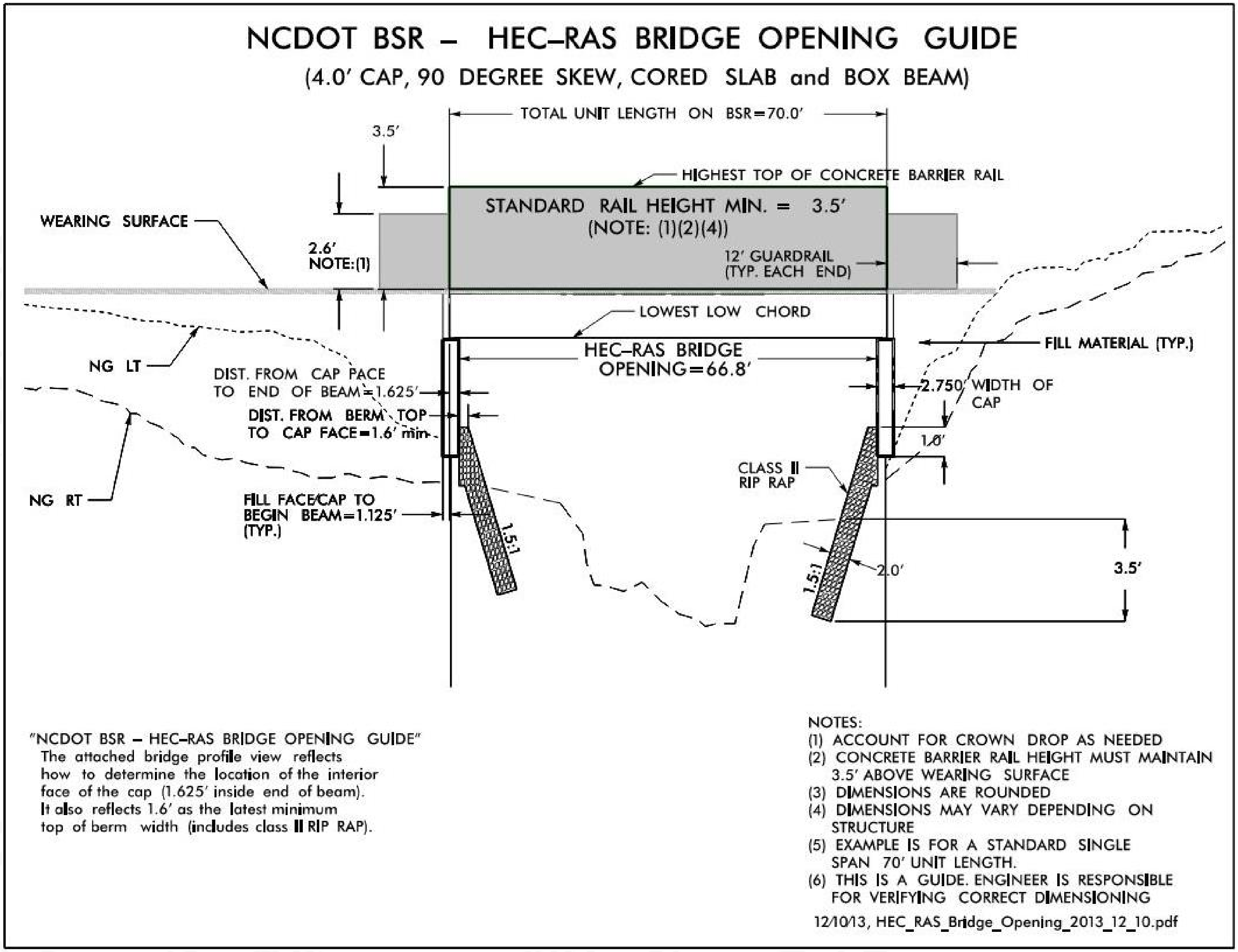

Figure 4. HEC-RAS Bridge Opening Guide (4.0' Cap)